CHAPTER 5

GENERAL CRITERIA FOR REINFORCED MASONRY

5-1. Introduction. This chapter provides the general criteria for the design of reinforced masonry using the working stress design method. Generally, a running bond masonry pattern is the basis of the design and reinforcing requirements contained herein. Running bond is the strongest bond pattern and will be used unless a stacked bond pattern is essential to the architectural treatment of the building. Additional design and detailing requirements for stacked bond masonry are contained herein.

5-2. Working stress assumptions. The assumptions for the working stress design of reinforced masonry are the same as the assumptions used in the working stress design of reinforced concrete.

a. Basic assumptions. The basic assumptions are as follows:

(1) Plane sections remain plane after bending.

(2) Stress is proportional to strain which is proportional to the distance from the neutral axis.

(3) The modulus of elasticity is constant throughout the member in the working load range.

(4) Masonry does not resist tension forces.

(5) Reinforcement is completely bonded so that the strain in the masonry and the strain in the reinforcement are the same at the location of the reinforcement.

(6) External and internal moments and forces are in equilibrium.

(7) The shearing forces are assumed uniformly distributed over the cross section.

b. Modular ratio. As per the basic assumptions above, the strain in masonry, ϵ_m, at a given load is equal to the strain in the reinforcing steel, ϵs, at the same location.

$$\epsilon_m = \frac{f_m}{E_m} = \epsilon_s = \frac{f_s}{E_s} \qquad \text{(eq 5-1)}$$

Where:

f_m = The stress in the masonry, psi.
f_s = The stress in the steel, psi.
E_m = The modulus of elasticity of masonry, psi.
E_s = The modulus of elasticity of steel, psi.

The modular ratio, n, is given by the following equation.

$$n = \frac{E_s}{E_m} \qquad \text{(eq 5-2)}$$

The relationship between f_s and f_m is then,

$$f_s = n(f_m) = \frac{E_s}{E_m}(f_m) \qquad \text{(eq 5-3)}$$

c. Transformed sections. When a masonry member is subjected to bending, the masonry above the neutral axis of the cross section is in compression. The masonry below the neutral axis is assumed cracked. The transformed section consists of the area of masonry above the neutral axis and n times the reinforcing steel area below the neutral axis. The transformed area of steel in tension, A_{trans}, is—

$$A_{trans} = (n)(As) \qquad \text{(eq 5-4)}$$

When the reinforcement and surrounding masonry is in compression, such as a column with a concentric axial load, A_{trans} is one of the following—

(1) For long term loading conditions;

$$A_{trans} = (2n - 1)A_s \qquad \text{(eq 5-5)}$$

(2) For other than long term loading conditions;

$$A_{trans} = (n - 1)A_s \qquad \text{(eq 5-6)}$$

Using n-1 or 2n - 1, rather than n, accounts for the area of masonry in compression being occupied by the actual steel area.

5-3. Structural properties. The structural properties of hollow concrete masonry units provided in this manual are based on the minimum dimensions given in ASTM C 90. These properties may also be assumed for hollow brick masonry with the same minimum dimensions.

a. Unit types. It is recommended that open-end units, as shown in figure 5-1, be used in all masonry construction. The open-end unit shown in figure 5-1 meets the requirements of ASTM C 90. The use of

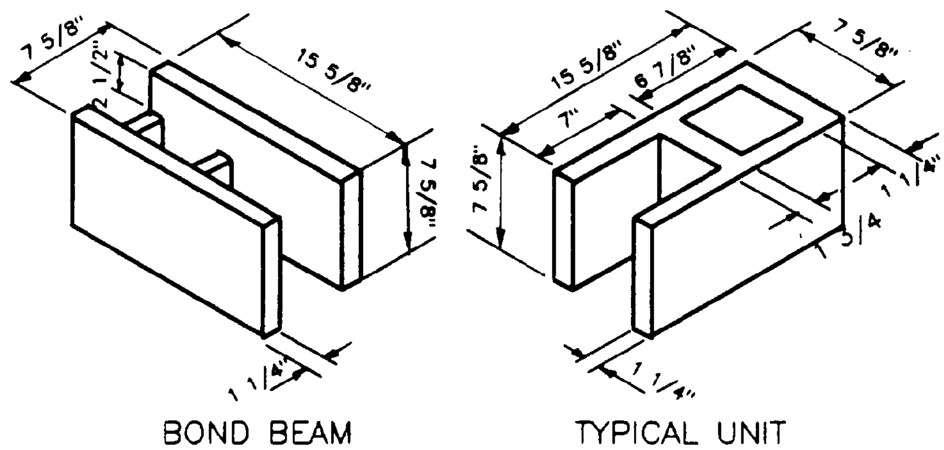


Figure 5–1. Open end unit 8 in X 8 in X 16 in.

open-end units allows placement of the vertical reinforcing steel with a minimum number of splices—thus vertical reinforcement can usually be continuous between supports. The vertical alignment of webs in open-end units provide large open cells that can be easily grouted. The grouted open-end cells provide good load transfer and allow for complete grouting of lintels and beams. Masonry units with three webs often have concave ends which makes it difficult to fully grout a wall. Therefore, when three web units are used in lintels, masonry beams, and fully grouted walls, grouting at each course level is required.

b. *Section properties of reinforced masonry.*

(1) *Assumed concrete masonry unit dimensions.* As a general rule, the dimensions for hollow CMU may be assumed as shown in figure 5-2 and given in table 5-1. These values will vary with unit type, geographic location, and manufacturer; however; they are considered conservative—thus were used in the design calculations in this manual.

Table 5–1. Assumed dimensions of hollow concrete masonry units and associated dimensions, inches.

CMU NOMINAL THICK.	CMU DESIGN THICK.	FACE SHELL THICK. (T_s)	WEB THICK. (t_w)	(d_1)	(d_2)	(b_w)
6	5⅝	1	1	2.81	—	7½
8	7⅝	1¼	1	3.81	5.31	7½
10	9⅝	1⅜	1⅛	4.81	7.06	7½
12	11⅝	1½	1⅛	5.81	8.81	7½

(2) *Equivalent wall thickness.* The equivalent thickness for masonry walls with hollow units and varying grouted cell spacings will be as shown in table 5-2.

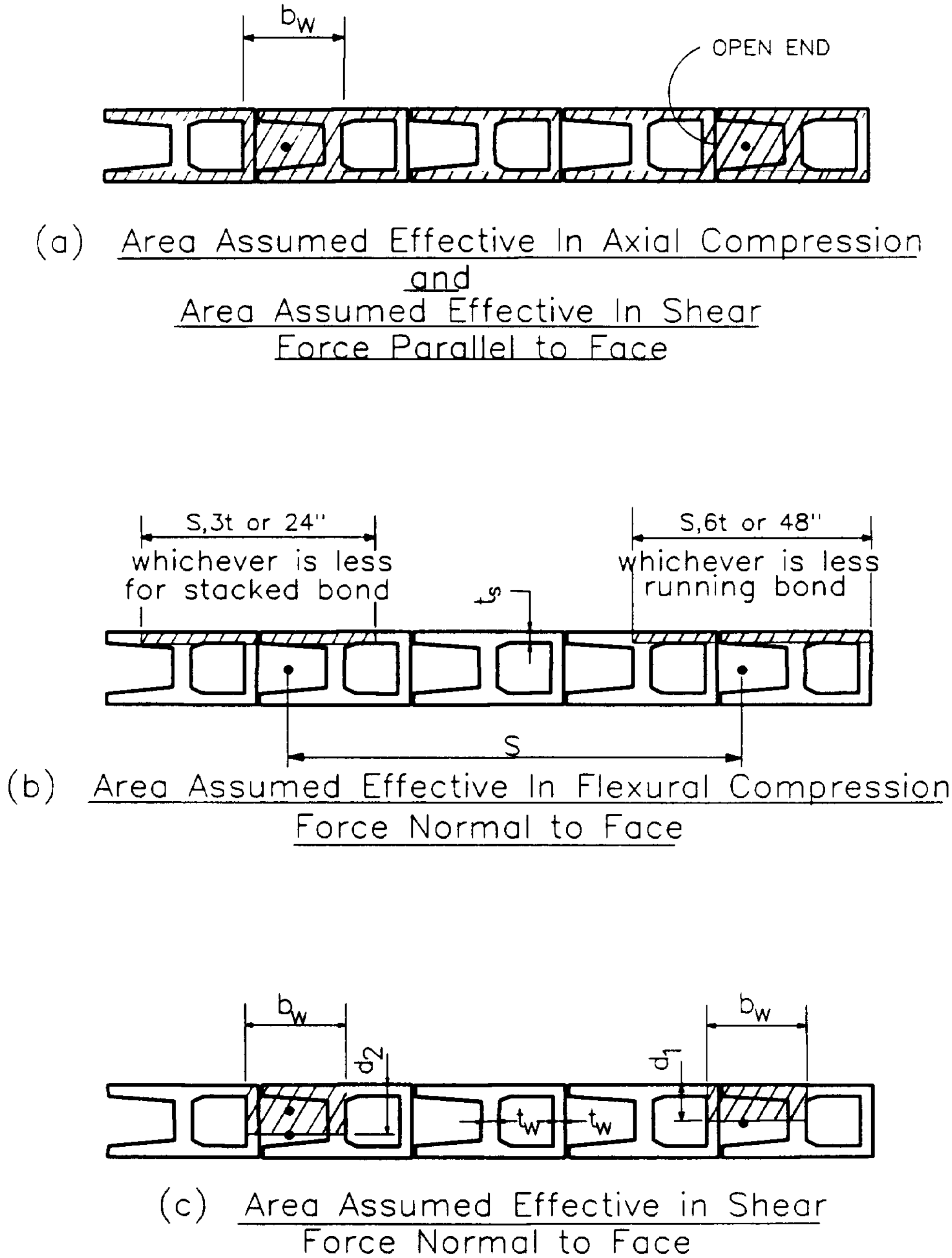
Table 5–2. Equivalent wall thickness for computing compression and shear stress parallel to the wall for hollow concrete masonry units, inches.[1]

SPACING OF GROUTED CELLS S, inches	NOMINAL WALL THICKNESS			
	6	8	10	12
Fully Grouted	5.62	7.62	9.62	11.62
16	3.70	4.90	5.98	7.04
24	3.13	4.10	4.91	5.70
32	2.85	3.70	4.37	5.02
40	2.68	3.46	4.05	4.62
48	2.57	3.30	3.83	4.35
56	2.49	3.19	3.67	4.16
64	2.42	3.10	3.56	4.01

Table 5-2. Equivalent wall thickness for computing compression and shear stress parallel to the wall for hollow concrete masonry units, inches.[1]—Continued

SPACING OF GROUTED CELLS S, inches	NOMINAL WALL THICKNESS			
	6	8	10	12
72	2.38	3.03	3.47	3.90
No Grout	2.00	2.50	2.75	3.00

[1]Based on face shells plus one 7½ inch wide web per "S" spacing. See figure 5-2a.



U. S. ARMY CORPS OF ENGINEERS

Figure 5-2. Assumed dimensions and effective areas of hollow masonry.

(3) *Effective area.* The effective area of hollow masonry used in design vary and are generally dependent upon the thickness of the face shells and the cross-webs, the width of grouted core, and the type of mortar bedding used in construction. Since contractors may use standard two-hole (plain or concave ends) or open-end concrete masonry units, and since exact configuration may vary between manufacturers, the precise effective area will be unknown at the time of design. The assumed effective areas for different loading conditions will be as illustrated in figure 5-2. Effective areas for masonry walls loaded in compression or in shear parallel to the wall are given in table 5-3. The effective area will be adjusted to reflect loss of area resulting from the use of reglets, flashing, slip-joints, and raked mortar joints.

Table 5–3. Area effective in axial compression and in in-plane shear, A_e' in^2/ft.[1]

SPACING OF GROUTED CELLS S, INCHES	NOMINAL WALL THICKNESS			
	6	8	10	12
Fully Grouted	68	92	116	140
16	44	59	72	85
24	38	49	59	68
32	34	44	52	60
40	32	42	48	55
48	31	40	46	52
56	30	38	44	50
64	29	37	43	48
72	28	36	42	47
No Grout	24	30	33	36

[1] Based on face shells plus one 7½ inch wide web per "S" spacing. See figure 5–2a.

(4) *Effective width of flexural compression block.* The effective width of the flexural compression stress block of reinforced masonry placed in both running and stacked bond patterns will be as illustrated in figure 5-2. The effective width will not exceed the spacing of the reinforcement, S.

(5) *Cracking moment and gross moment of inertia.* The cracking moment strength of a wall, M_{cr}, will be determined as follows:

$$M_{cr} = \frac{2I_g f_r}{t} \text{ (1b-in)} \qquad \text{(eq 5-7)}$$

Where:

f_r = the modulus of rupture for calculating deflection equal to $2.5\sqrt{f'_m}$, $2I_g/t$ is the section modulus, psi.

t = The actual thickness of the wall, inches.

I_g =The gross moment of inertia of the wall, in^4.

The cracking moment strength along with the gross moment of inertia, I_g, are listed in table 5-4 for various wall thicknesses and reinforcing spacing.

Table 5–4. Gross moment of inertia and cracking moment strength for various widths of CMU walls[1]. *Type S mortar, f'_m = 1350 psi.*

WIDTH	NOMINAL WALL THICKNESS							
	6		8		10		12	
b^2 in	I_g in^4	M_{cr} ft-lb	I_g in^4	M_{cr} ft-lb	I_g in^4	M_{cr} ft-lb	I_g in^4	M_{cr} ft-lb
48	—	—	1319	2648	2470	3929	4119	5424
40	—	—	1113	2235	2092	3328	3499	4608
32	377	1027	907	1822	1714	2727	2879	3792
24	290	791	702	1409	1337	2126	2260	2976
16	204	554	496	995	959	1525	1640	2160
8	119	323	296	593	594	946	1047	1379

[1]Based on face shells plus one 7½ inch wide web per "S" spacing. See figure 5–2a.
[2]"b" is assumed to be "S". It is limited to 6 times the nominal wall thickness, but not more than 48 inches. See figure 5–2b.

(6) *Design aids.* Section properties of reinforced masonry with type S and N mortars are given in appendix B, tables B-1 through B-14.

c. Weight of masonry. The design examples and tables included in this manual are based on normal-weight hollow masonry units. Normal-weight units are assumed to have an oven-dry weight of concrete of 145 pounds per cubic foot. Weights of lighter-weight units may be obtained by direct proportion to the lighter weight of concrete being used. Table 5-5 gives the weight of concrete masonry unit walls.

Table 5–5. Weight of CMU walls[1], w_2, pounds per square foot.

SPACING OF GROUTED CELLS, S, inches	NOMINAL WALL THICKNESS			
	6	8	10	12
Fully Grouted	68	92	116	140
16	58	75	92	111
24	53	69	85	102
32	51	65	78	93
40	50	62	75	89
48	49	60	72	85
56	48	58	70	83
64	47	57	69	81
72	46	56	68	80
No Grout	43	50	59	69

[1]Based on normal-weight units having a concrete weight of 145 pounds per cubic foot. An average amount has been added into those values to include the weight on bond beams and reinforcing.

Table 5-6 gives the average weights, gross and net areas of concrete masonry units.

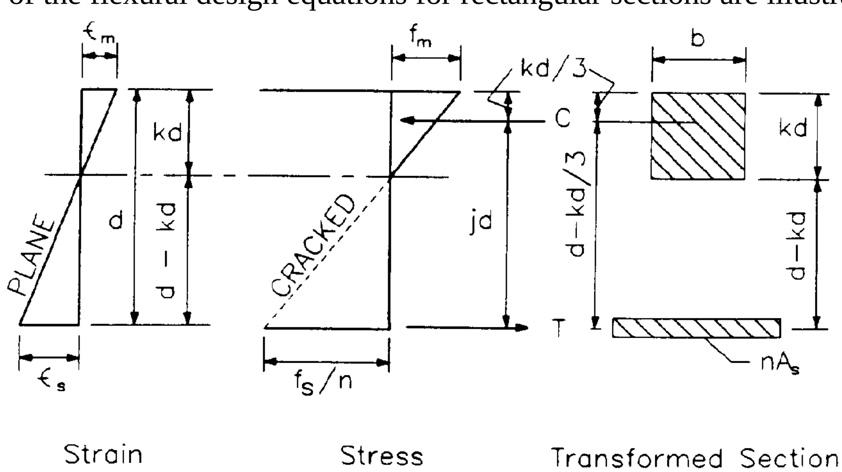
Table 5–6. Gross areas, net areas and average weights of concrete masonry units [1].

THICKNESS (in)	GROSS AREA OF UNIT (in^2)	NET AREA OF UNIT (in^2)	LIGHT-WEIGHT AGGREGATE [2] (lbs/unit)	SAND-GRAVEL AGGREGATE [2] (lbs/unit)
4	57	37	15	20
6	88	50	23	33
8	119	57	28	38
12	182	83	40	56

[1]The values given in this table are average values. Actual values will vary with type of unit and manufacturer. However, these table values will normally be sufficient for estimating purposes.
[2]Light-weight units are assumed to have a concrete weight of 105 pcf and sand-gravel units a concrete weight of 145 pcf.

5-4. Working stress design equations. The equations in this paragraph are the basic working stress equations used in the design of reinforced masonry.

a. Flexural design rectangular sections. The design assumptions, coefficients and cross section geometry used in the derivation of the flexural design equations for rectangular sections are illustrated in figure 5-3.



U. S. ARMY CORPS OF ENGINEERS

Figure 5–3. Working stress flexural design assumptions for rectangular sections.

(1) *Design coefficients.* In the design of reinforced rectangular sections, the first step is to locate the neutral axis. This can be accomplished by determining the coefficient, k, which is the ratio of the depth of the compressive stress block to the total depth from the compression face to the reinforcing steel, d. k is derived by equating the moment of the transformed steel area about the centroidal axis of the cross section to the moment of the compression area about the centroidal axis as follows:

kd

$$b(kd)\left[\frac{kd}{2}\right] = nA_s\,(d - kd)$$

Rearranging;

$$\frac{b(kd)^2}{2} - nA_s\,(d - kd) = 0$$

The steel ratio, p, is determined by:

$$p = A_s/bd \qquad \text{(eq 5-8)}$$

Substituting phd for A_s;

$$\frac{b(kd)^2}{2} - npbd(d - kd) = 0$$

Dividing through by bd^2;

$$\frac{k^2}{2} - pn(1 - k) = 0$$

From which;

$$k = \left[\,(np^2 + 2np\,\right]^{1/2} - np \qquad \text{(eq 5-9)}$$

The coefficient j, which is the ratio of the distance between the resultant compressive force and the centroid of the tensile force to the distance d, is determined by—

$$j = 1 - \frac{k}{3} \qquad \text{(eq 5-10)}$$

The balanced steel ratio in the working stress design method, p_e, is defined as the reinforcing ratio where the steel and the masonry reach their maximum allowable stresses for the same applied moment. p_e is determined by the equation 5-11 as follows:

$$p_e = \frac{n}{2(F_s/F_m)\,[n + F_s/F_m)]} \qquad \text{(eq 5-11)}$$

Where:

F_s = The allowable tensile stress in the reinforcing steel, psi.
F_m = The allowable flexural compressive stress in the masonry, psi.

(2) *Computed working stresses.* The working stresses for the steel and the masonry are computed as follows:

(a) If $p < p_e$, the steel stress, f_s, will reach its allowable stress before the masonry and equation 5-12 will control.

$$f_s = \frac{M}{A_s jd}(\text{psi}) \qquad \text{(eq 5-12)}$$

(b) If $p > p_e$, the masonry stress, f_m, will reach its allowable stress before the steel and equation 5-13 will control.

$$f_m = \frac{2M}{kjbd^2}(\text{psi}) \qquad \text{(eq 5-13)}$$

Where:

M = The moment, inch-kips.
b = The width of the member effective in compression as shown in figure 5-2b, inches.

(3) *Resisting moments.*

(a) The resisting moment for the reinforcement, M_{rs}, can be determined by substituting the allowable steel stress, F_s, for the computed steel stress in equation 5-12 and solving for the moment.

$$M_{rs} = \frac{F_s A_s jd}{12}(\text{ft-lb}) \qquad \text{(eq 5-14)}$$

(b) The resisting moment for masonry, M_{rm}, can be determined by substituting the allowable masonry stress, F_m, for the computed masonry stress in equation 5-13 and solving for the moment.

$$M_{rm} = \frac{F_m kjbd^2}{2(12)} (ft-lb) \quad \text{(eq 5-15)}$$

b. Flexural design T-sections. The coefficients and cross section geometry used in the derivation of the flexural design equations for T-sections are illustrated in figure 5-4.

(1) *Design coefficients.* In the design of reinforced T-sections, as in the design of rectangular sections, the first step is to locate the neutral axis. As with rectangular sections, this can be accomplished by determining the coefficient k_T. k_T is derived by assuming that the compressive force in the flange, C, is equal to the tension force in the reinforcement. The contribution of the portion of the web in compression is small and can be neglected, therefore if;

$$T \approx C$$

Then,

$$pbdf_s = f_m \left[\frac{2k_T d - t_s}{2k_T d}\right] bt_s \quad \text{(eq 5-16)}$$

Where:

t_s = The thickness of the face shell of the unit, inches.

From the strain compatibility relationship it can be determined that;

$$k_T = \frac{n}{n + (f_s/f_m)}$$

Rearranging the equation and solving for fm yields:

$$f_m = (fs) \frac{k_T}{n(1 - k_T)}$$

Substituting this equation for fm into equation 5-16 yields;

$$k_T = \frac{np + 1/2(t_s/d)^2}{np + (t_s/d)} \quad \text{(eq 5-17)}$$

The coefficient, j_T, can be determined by the relationship,

$$j_T d = d - z$$

Where:

z = The distance from the extreme compressive fiber to the center of compression (or the center of gravity of the trapezoid shown in figure 5-4) and is determined as follows:

$$z = \left[\frac{3k_T d - 2t_s}{2k_T d - t_s}\right]\left[\frac{t_s}{3}\right]$$

From the above equations j_T can be determined by:

$$j_T = \frac{6 - 6(t_s/d) + 2(t_s/d)^2 + (t_s/d)^3[1/(2pn)]}{6 - 3(t_s/d)} \quad \text{(eq 5-18)}$$

The resisting moments of the steel and masonry are equal to the product of the moment arm, $j_T d$, and the tension or compression force, respectively, therefore:

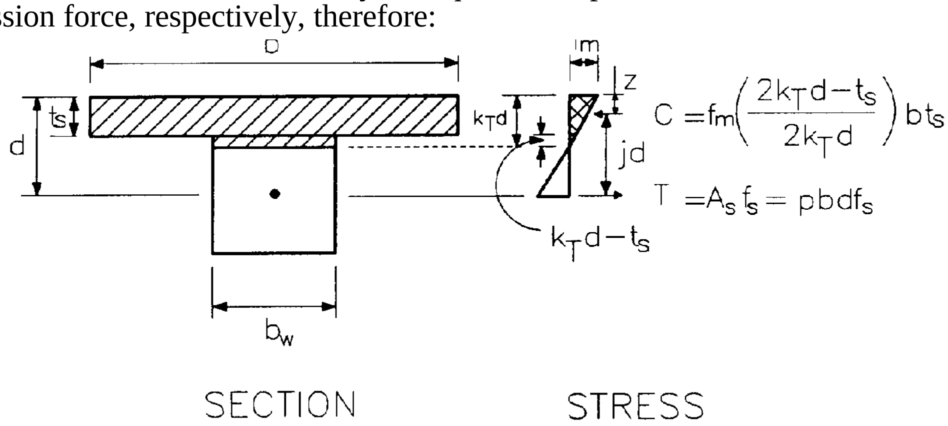


Figure 5-4. Working stress flexural design assumptions for T-sections.

$$M_{rs} = \frac{F_s A_s jd}{12}(\text{ft}-\text{lb}) \qquad \text{(eq 5-19)}$$

And,

$$M_{rm} = \frac{F_m kjbd^2}{12}\left[1 - \frac{t_s}{2k_T d}\right](\text{ft}-\text{lb}) \qquad \text{(eq 5-20)}$$

c. Design for axial compression.

(1) When determining the capacity of a masonry wall element in compression, the compression reinforcement in the element will be neglected since its contribution is not significant. Only tied compression reinforcement, such as in a column or pilaster will be considered effective. The axial stress in a masonry wall, fa, is found as follows:

$$f_a = \frac{P}{A_e}(\text{psi}) \qquad \text{(eq 5-21)}$$

Where:

P = The axial load, lbs.

A_e = The area of the element effective in compression as shown in figure 5-2a and obtained from table 5-3, in^2.

(2) The design of columns in axial compression is given in chapter 9.

d. Design for shear.

(1) For shear design in masonry walls subjected to out-of-plane loading, the shear stress in a masonry element, f_v, is found as follows:

$$f_v = \frac{V}{b_w d} \qquad \text{(eq 5-22)}$$

Where:

V = The shear load, lbs.

b_w = The width of the masonry element effective in resisting out-of-plane shear as shown in figure 5-2c and given in table 5-1, inches.

d = The depth of the masonry element effective in resisting the shear is shown in figure 5-2c and given in table 5-1, inches. For one bar per cell, $d = d_1$, and for two bars per cell, $d = d_2$.

(2) For shear design in masonry walls subject to in-plane loading (shear walls) the shear stress in a masonry element is found as follows:

$$f_v = \frac{V}{A_e} \qquad \text{(eq 5-23)}$$

Where:

V = The shear load, lbs.

A_e = The area masonry element effective in resisting in-plane shear as shown in figure 5-2a and obtained from table 5-3, inches.

5-5. Allowable working stresses. The allowable working stresses for masonry, F_m, (CMU and brick) are given in table 5-7. These allowables are based on the masonry compressive strength, f'_m, which either have been assumed or have been determined from prism tests. The assumed f'_m values, 1500 psi for solid units and 1350 psi for hollow units, are for type M and type S mortars. If type N mortar is used, f'_m will be assumed to be 1000 psi for all units. The assumed f'_m values are reasonable and conservative in that they are in the range ⅓ to ¾ of the prism strength. If the designer needs to use higher masonry strengths than the assumed values, prism tests may be required. Generally reinforced masonry will be designed and detailed in conformance with the assumed values given in table 5-7.

TYPE OF STRESS	SOLID UNITS	HOLLOW UNITS[2]	SOLID AND HOLLOW UNITS	
For Grades of Materials Specified[4]	f'_m = 1,500 psi[3]	f'_m = 1,350 psi[3]	f'_m	
	Building Brick: ASTM C62, Grade MW or SW Facing Brick: ASTM C216, Grade MW or SW Concrete Building Brick: ASTM C55 Type 1 Concrete Masonry Units: ASTM C90 Type 1	Concrete Masonry Units: ASTM C90, Type 1 Glazed Structural Facing Units: ASTM C126 Type I Hollow Brick Unit: ASTM C652 Grade MW or SW	For materials where ultimate compressive stress (f'_m) is established by approved prism tests, but not to exceed 3,500 psi.	But Not To Exceed
COMPRESSION:				
Axial, Walls, F_a	Equation 5-24	Equation 5-24		Equation 5-24
Axial, Columns, F_a[a]	Equation 9-1	Equation 9-1		Equation 9-1
Flexural, F_b	500	450	$1/3\ f'_m$	900
SHEAR:				
No Shear Steel:[5]	39	37	$1.0\sqrt{f'_m}$	50
Full Shear Steel:[6]				
Flexural Members	117	111	$3.0\sqrt{f'_m}$	120
Shear Walls	Equations 7-1 thru 7-4	Equations 7-1 thru 7-4	Equations 7-1 thru 7-4	
MODULUS:				
Elasticity	1,500,000	1,350,000	$1000f'_m$	3,000,000
Rigidity	600,000	540,000	$400f'_m$	1,200,000
BEARING:				
On Full Area	375	338	$.25f'_m$	900
On 1/3 or less of Area[7]	450	405	$.30f'_m$	1,050

[1]All allowable stresses will be increased one-third when wind or seismic forces are included, provided the required section or area computed on this basis is not less than that required without wind or seismic forces.

[2]Stresses will be based on the net section. Figure 5-3 applies.

[3]Where prism tests are not performed these values of f'_m may be assumed for types M and S mortar when the units comply with the applicable ASTM standard. If type N mortar is used f'_m will be assumed to be 1000 psi for all units.

[4]Minimum compressive strength at 28 days for grout and mortar will be as follows: Grout = 2000 psi, Type S mortar = 1800 psi, Type M mortar = 2500 psi and Type N mortar = 750 psi.

[5]Web reinforcement will be provided to carry the entire shear in excess of 20 psi whenever there is required negative reinforcement and for a distance of one-sixteenth the clear span beyond the point of inflection.

[6]Reinforcement must be capable of taking the entire shear.

[7]This increase will be permitted only when the edges of the loaded and unloaded area is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third but less than the full area will be interpolated between the values given.

Table 5-7. Allowable working stresses in reinforced masonry[1].

The stresses in the reinforcing steel will not exceed the values shown in table 5-8.

Table 5-8. Allowable working stresses for Grade 60 reinforcing bars.

TYPE OF STRESS	PSI
Tensile	24,000
Compressive	24,000

The allowable axial stress, Fa, can be determined as follows:

$$F_a = 0.20 f'_m R \qquad \text{(eq 5-24)}$$

Where:

R = The stress reduction factor for the wall based on the height to thickness ratio as follows:

$$R = \left[1 - \left[\frac{12h}{40t_n}\right]^3\right] \qquad \text{(eq 5-25)}$$

Where:

h = The clear height of the wall, feet.

t_n = The nominal thickness of the wall, inches.

The stress reduction factor limits the axial stress on the wall so that buckling will not occur. When analyzing the top or bottom of a wall, where buckling is not a concern, the stress reduction factor should not be used.

5-6. Basic reinforcement requirements. The design of steel reinforcing bars will be based on the working stress allowables given in table 5-8.

a. Minimum bar size. The minimum bar size will be No. 4.

b. Maximum bar sizes. The most commonly used, and preferred, reinforcing steel bar sizes in CMU walls are Nos. 4, 5 and 6. When the design requires the use of larger bars, the bar size will not exceed No. 6 bars in 6-inch CMU walls, No. 7 bars in 8-inch CMU walls and No. 8 bars in 10-inch and 12-inch CMU walls. This provides reasonable steel ratios, reasonable splice lengths, and better distribution of reinforcement. The maximum bar size in masonry columns should be No. 9.

c. Maximum flexural reinforcement. There is no maximum flexural reinforcement limit in the working stress design method, however there is a practical maximum. It is not efficient to use a steel ratio, p, that is greater than the balanced-stress steel ratio, p_e. Examining the elastic theory shows that reinforcing steel added to a masonry element with $p > p_e$ provides less than one half the added strength the same amount of steel added to the member with $p < p_e$ provides. Although using p > pe is not efficient use of the reinforcement, in some instances it may be more economical from a total wall cost standpoint to increase the reinforcement in lieu of increasing the wall thickness. Thus, the decision to use more than balanced steel becomes an economic one and should be decided on a case by case basis.

(1) Table 5-9 lists values of p_e, k and j for varying values of f'm.

Table 5-9. Balanced reinforcing steel ratio along with k and j for fully grouted CMU in running bond. f_y = 60,000 psi.

f'_m	p_e	k	j
1350	0.0027	0.287	0.904
1500	0.0030	0.287	0.904
2000	0.0040	0.287	0.904
2500	0.0050	0.287	0.904

(2) Table 5-10 may be used by designers to determine the bar size and spacing that will achieve a near balanced-stress ratio for varying wall thicknesses with one bar per cell. The table also provides the depth to the reinforcement, d; the balance-stress steel ratio, p_e the actual steel ratio, p, and the actual depth of the compression stress block, kd; using the respective bar size and spacing.

Table 5-10. Balanced reinforcing steel, one bar per cell, fully grouted CMU walls [1] in running bond. See figure 5-2 for maximum effective width. f'_m = 1350 psi, fy = 60,000 psi.

CMU THICK.[2]	d	p_e	Reinf. ≈ p_e	Actual p	Actual k	Actual kd
6	2.81	0.0027	#4 @ 24″	0.0030	0.300	0.84″
			#5 @ 40″[3]	0.0028	0.292	0.82″
8	3.81	0.0027	#4 @ 24″	0.0022	0.264	1.01″
			#5 @ 32″	0.0025	0.278	1.06″
			#6 @ 48″	0.0024	0.274	1.04″
10	4.81	0.0027	#4 @ 16″	0.0026	0.283	1.36″
			#5 @ 24″	0.0027	0.287	1.38″
			#6 @ 32″	0.0029	0.296	1.42″
			#7 @ 48″	0.0026	0.283	1.36″
12	5.81	0.0027	#4 @ 16″	0.0022	0.253	1.47″
			#5 @ 24″	0.0027	0.287	1.67″
			#6 @ 32″	0.0024	0.274	1.59″
			#7 @ 40″	0.0026	0.283	1.64″
			#8 @ 48″	0.0028	0.292	1.70″

[1]When the walls are partially grouted, the design section will sometimes be a T-beam, however, the difference is usually not significant.
[2]Masonry unit thicknesses are nominal.
[3]Note that this spacing exceeds the maximum effective width of six times the nominal wall thickness given in figure 5-2b.

(3) Table 5-11 provides similar information to that given in table 5-10 for CMU with two bars per cell.

Table 5–11. Balanced reinforcing steel, two bars per cell, fully grouted CMU walls [1]. f'_m = 1350 psi, fy = 60,000 psi

CMU THICK.[2]	d	p_e	Reinf. ≈ p_e	Actual p	Actual k	Actual kd
8	5.31	0.0027	#4 @ 16″	0.0024	0.274	1.45″
			#5 @ 24″	0.0024	0.274	1.45″
			#6 @ 32″	0.0026	0.283	1.50″
			#7 @ 40″	0.0028	0.292	1.55″
10	7.06	0.0027	#4 @ 16″	0.0018	0.242	1.71″
			#5 @ 16″	0.0027	0.287	2.03″
			#6 @ 24″	0.0026	0.283	2.00″
			#7 @ 32″	0.0027	0.287	2.03″
			#8 @ 40″	0.0028	0.292	2.06″
12	8.81	0.0027	#5 @ 16″	0.0022	0.264	2.33″
			#6 @ 24″	0.0021	0.259	2.28″
			#7 @ 24″	0.0028	0.292	2.57″
			#8 @ 32″	0.0028	0.292	2.57″

[1]When the walls are partially grouted, the design section will be a T-beam, however, the difference is usually not significant.

[2]Masonry unit thicknesses are nominal.

(4) Tables 5-9, 5-10 and 5-11 are applicable to partially and fully grouted CMU walls, as stated. In partially grouted CMU walls, when the stress block falls below the face shell, that is, when kd > t_s (given in table 5-1), the tables do not apply. With kd > t_s, the design will be based on the T-section design method contained herein. In most cases, when one bar is used per cell, the stress block falls within the face shell and when two bars are used per cell, the stress block falls outside the face shell. However, for CMU with f'_m = 1350 psi, the difference between a T-section beam design and a rectangular section beam design is usually so insignificant that a rectangular beam design will suffice. When providing two bars per cell, the vertical bars will be placed outside the horizontal reinforcement. Details will be provided on the drawings showing this relationship so that the depth to reinforcement assumed in design will be provided during construction. The details should also provide adequate space to allow grout to be placed and vibrated.

d. Minimum reinforcement. All masonry exterior, bearing and shear walls (structural walls) will be reinforced as provided below. There are no minimum reinforcement requirements for nonstructural partitions except around openings as given in chapter 4. In seismic zones 1 through 4, the minimum reinforcement requirements given in TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13 must also be satisfied.

(1) One vertical reinforcing bar will be provided continuously from support to support at each wall corner, at each side of each opening, at each side of control joints, at ends of walls, and elsewhere in the wall panels at a maximum spacing of six feet. This minimum reinforcement will be the same size as the minimum vertical reinforcement provided for flexural stresses.

(2) Horizontal reinforcement will be provided continuously at floor and roof levels and at the tops of walls. Horizontal reinforcement will also be provided above and below openings. These bars will extend a minimum of 40 bar diameters, but not less than 24 inches, past the edges of the opening. For masonry laid in running bond, the minimum horizontal reinforcement should be one No. 4 bar per bond beam. For masonry laid in other than running bond, such as stacked bond, the minimum area of horizontal reinforcement placed in horizontal joints or in bond beams, which are spaced not more than 48 inches on center, will be 0.0007 times the vertical cross sectional area of the wall. Lintel units will not be used in lieu of bond beam units, since lintel units do not allow passage of the vertical reinforcement. If the wall is founded on a concrete foundation wall, the required reinforcement at the floor level may be provided in the top of the foundation wall.

e. Splices of reinforcement. The length of tension and compression lap splices will be $48d_b$, where d_b is the diameter of the bar. All other requirements for the development and splices of reinforcement will be in accordance with ACI 530/ASCE 5.

5-7. Connections between elements. Great care must be taken to properly design and detail connections between the vertical resisting elements (masonry shear walls) and the horizontal resisting elements (diaphragms) of the building so that all elements act together to provide an integral structural system. A positive means of connection will be provided to transfer the diaphragm shear forces into the shear walls. In designing connections or ties, it is necessary to trace the forces through their load paths and also to make every connection along each path adequate and consistent with the basic assumptions and distribution of forces. Because joints and connections directly affect the integrity of the structure, their design and fabrication must be adequate for the functions intended. In designing and detailing, it must be recognized that the lateral forces are not static, as assumed for convenience, but dynamic and to a great extent unpredictable. Because of this, it is important to provide the minimum connections required below even when they are not specifically required for design loading. In seismic zones 1 through 4 the minimum connection requirements given in TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13 must also be satisfied. When the design forces on joints and connections between lateral force resisting elements are due to wind, the minimum criteria given in TM 5-809-2/AFM 88-3, Chapter 2, will be followed.

a. Forces to be considered. Forces to be considered in the design of joints and connections are gravity loads; temporary erection loads; differential settlement; horizontal loads normal to the wall; horizontal loads parallel to the wall; and creep, shrinkage, and thermal forces; separately or combined as applicable. Bond beams at roof or floor diaphragm levels must have the reinforcement continuous through control joints to resist the tensile and compressive chord stresses induced by the diaphragm beam action. The connections between the diaphragm and chord (bond beam) members must be capable of resisting the stresses induced by external loadings.

b. Joints and connections. Joints and connections may be made by welding steel reinforcement to structural steel members, by bolting, by dowels, by transfer of tensile or compressive stresses by bond of reinforcing bars, or by use of key-type devices. The transfer of shear may be accomplished by using reinforcing steel extended as dowels coupled with cast-in-place concrete placed between roughened concrete interfaces or by mechanical devices such as embedded plates or shapes. The entire shear loading should be transferred through one type of device, even though a combination of devices may be available at the joint or support being considered. Maximum spacing of dowels or bolts, for load transfer between elements, will not exceed four feet. All significant combinations of loadings will be considered, and the joints and connections will be designed for forces consistent with all reasonable combinations of loadings as given in TM 5-809-1/AFM 88-3, Chapter 1. Details of the connections will be based on rational analysis in accordance with established principles of mechanics.

c. Allowable tension and shear on bolts. The allowable loads for plate, headed, and bent bar anchor bolts embedded in masonry will be determined in accordance with the criteria in ACT 530/ASCE 6. Tables 5-12, 5-13 and 5-14 were developed using that criteria.

Table 5-12. Allowable tension in bolts, B_a, in pounds, based on the compressive strength of masonry.

	1_b [1] or 1_{be} [2], inches [3]							
f'_m psi	2	3	4	6	7	8	9	10
1350	230	520	920	2080	2830	3690	4670	5770
1500	240	550	970	2190	2980	3890	4930	6080
2000	280	630	1120	2530	3440	4500	5690	7024
2500	310	710	1260	2830	3850	5030	6360	7850
3000	340	770	1380	3100	4220	5510	6970	8600

[1] 1_b is the embedment length of the bolt, as shown in figure 5-5. It shall not be less than 4 bolt diameters.

[2] 1_{be} is the edge distance, as shown in figure 5-5.

[3] When the spacing between bolts is less than two times 1_b, the allowable loads will be reduced in accordance with the requirements in ACI 530/ASCE 6. When 1_{be} is less than 1_b or the distance to an ungrouted cell, the allowable loads will be reduced in accordance with the requirements in ACI 530/ASCE 6.

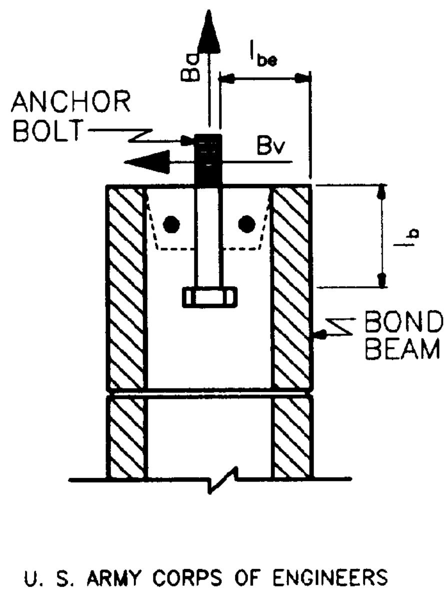


Figure 5–5. Effective embedment, l_b, and edge distance, l_{be}

Table 5–13. Allowable tension in bolts, B_a, in pounds, based on a steel yield strength of 36,000 psi.

BOLT DIAMETER, inches				
⅝	¾	⅞	1	1⅛
2210	3180	4330	5650	7160

Table 5–14. Allowable shear, B_v [1], in pounds, based on the listed value of f'_m and a steel yield strength of 36,000 psi.

	BOLT DIAMETER, inches				
f'_m psi	⅝	¾	⅞	1	1⅛
1350	1330	1730	1870	2000	2120
1500	1330	1780	1920	2050	2180
2000	1330	1910	2060	2200	2330
2500	1330	1910	2180	2330	2470
3000	1330	1910	2280	2440	2580

[1]This table is based on an edge distance of 12 bolt diameters or more. Where the edge distance is less than 12 bolt diameters the value of B_a will be reduced by linear interpolation to zero at an edge distance of 1½ inch. All bolts will be grouted in place with a minimum of 1 inch of grout between the bolt and the masonry.

d. Cautionary notes for designers and detailers. Avoid connection and joint details which would result in stress concentrations that might result in spalling or splitting of face shells at contact surfaces. To avoid stress concentrations, liberal chamfers, adequate reinforcement, and bearing pads should be used. Avoid direct bearing of heavy concentrated loads on face shells of concrete masonry units. Avoid welding to any embedded metal items which might cause spalling of the adjacent masonry, in particular where the expansion of the heated metal is restrained by masonry. All bolts and dowels which are embedded in masonry will be grouted solidly in place with not less than one inch of grout between the bolt or dowel and the masonry. Expansion anchors should not be used in the connection between major structural elements, including the connection of the horizontal elements (diaphragms) to the vertical elements (shear walls). At tops of piers and columns, vertical bolts will be set inside the horizontal ties. When steel beams are connected to masonry, the connection will allow for the thermal expansion and contraction of the beam. The construction case, where a wide range of temperatures can be expected if the beams are directly exposed to the heat of the sun, will be considered when determining the temperature differential.